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Parametric evaluation of racking performance of Platform timber framed walls

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Abstract

This paper provides a quantitative assessment of the racking performance of partially anchored timber framed walls, based on experimental tests. A total of 17 timber framed wall specimens, constructed from a combination of materials under different load configurations, were tested. The experimental study was designed to examine the influence of a range of geometrical parameters, such as fastener size and spacings, wall length, arrangement of studs and horizontal members, as well as the effect of vertical loading on the racking strength and stiffness of the walls. The experimental results were then compared with results obtained from design rules, as given in the relevant European standards, to determine the racking performance of the walls, and are discussed in the paper.

Keywords: Platform framing, Timber framed walls, Raking performance, Raking test, PD6693-1, Eurocode 5.

1. Introduction

Timber Platform frame construction is widely recognised as an effective and efficient building method for multi-storey buildings and in particular, in residential dwellings. In the Platform framing construction method, each wall is formed from individual wall units which can be constructed off-site, resulting in a reduction in on-site construction and associated costs, as well as achieving a higher quality of

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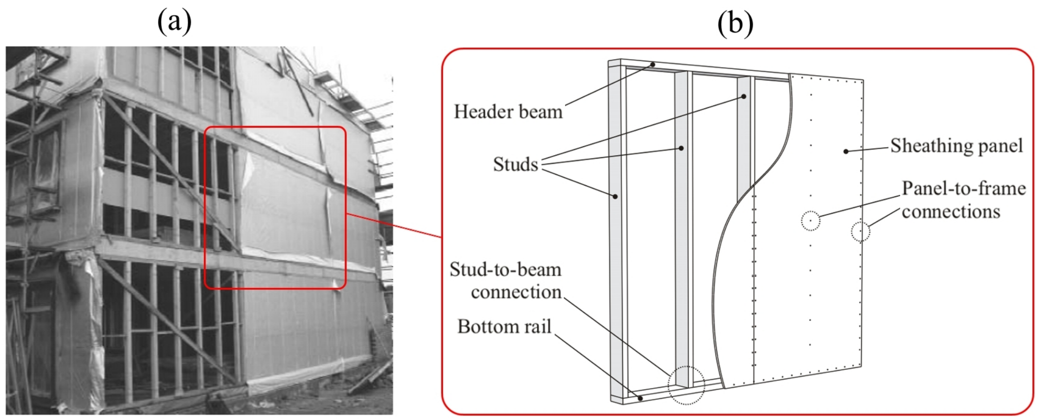

Figure 1: Platform construction method; (a) a continuous wall diaphragms is built by joining of pre-fabricated single unit walls; (b) schematic representation of a single unit wall.

the finished product. A wall unit is constructed from stud, beam and rail members, faced on one or both sides with a sheathing material and a typical example is shown in Figure 1-b. In the UK, for perimeter walls the sheathing material is typically: Oriented Strand Board, Particleboard or Plywood and is fastened to the stud, beam and rail members using nail or screw fixings.

Platform framed walls can be classified in two separate categories, according to the structural role they are designed for [1]:

- Stud walls: essentially intended for carrying vertical loading only. In such a case, sheathing panels (where used) only provide additional strength to the studs against in-plane and out-of-plane axial buckling.
- Racking walls: In addition to provide resistance to vertical loading, these walls are also designed to withstand in-plane lateral actions. This is achieved through the lateral strength of the fixings connecting the sheathing to the timber frame (i.e. studs, beam, rail members) as well as through shear buckling resistance of the sheathing material. The elements work as a system with the timber frame to provide racking stiffness and strength to the wall against lateral loading arising from the effects of wind or earthquake actions.

The body of past research work on the racking behaviour of timber framed walls is remarkably extensive. As pointed out by Källsner et al. [2], first research on the topic can be traced back to the late 1920's. Restricting the focus on (more recent) experimental-based approaches only, several fields of investigation and adopted testing methods can be identified. In particular, dynamic [3, 4, 5, 6] and cyclic load [7, 8, 9, 10, 11] testing: aimed at providing a better understanding and characterisation of the wall behaviour under the effect of seismic actions. Quasi-static monotonic testing, has been used to assess the influence of various parameters, such as the presence and size of openings [8, 9, 12], the type of fixing being used for the panel-to-frame connections (i.e. nails or staples [13]) and the use of reinforcements [14, 15]. Recently, the permanent reduction in mechanical properties of timber framed walls due to flooding has also been experimentally investigated [16].

1.1. Fully and partially anchored racking walls

In timber frame construction, racking walls are often classified in two categories: fully anchored and partially anchored walls. Fully anchored walls are walls which are prevented from lifting, when subjected to a lateral load, by the use of anchors (such as steel brackets) secured to underlying support structure or by the weight/actions the wall supports. For partially anchored walls, resistance against lifting is provided solely by the fixings between the sheathing and the bottom rail and fixings between the bottom rail connection to the support structure. Because of the absence of holding down ties in partially anchored walls, the studs experience a moderately high amount of uplift when the wall is subjected to in-plane racking loads.

1.2. Research aims

In the UK, the most common form of racking wall used in Platform timber construction is the partially anchored wall, and the experimental study covered in this paper focuses on this method of construction, with the main aim of evaluating the influence of a range of geometrical parameters (and configurations that replicate typical construction practices) on the racking strength and stiffness of the walls. The main focus of the research has been to determine the effects of parameters such as:

- panel-to-frame fastener spacings,
- wall length,

- arrangement and composition of studs and bottom rail members (e.g. use of double studs and double bottom rail),
- magnitude of vertical loading

on the racking performance of OSB sheathed walls. The study has also aimed to assess the differences between the experimental results and the design racking values obtained from the relevant European standards, in particular, the requirement of the UK National Annex to Eurocode 5 (EC5) [17], regarding the design for racking strength of timber framed walls using the procedure described in the PD 6693-1 document [18].

2. Method

In order to fulfil the aforementioned aims, an experimental test programme was designed and carried out. Descriptions of the tested wall specimens, and an outline of the adopted test set up and test series, are provided in subsections 2.1 to 2.3, with a brief description of the analytical method used based on PD 6693-1, to calculate the racking strength values, given in section 2.5. Finally, the experimental and analytical results are discussed in section 3.

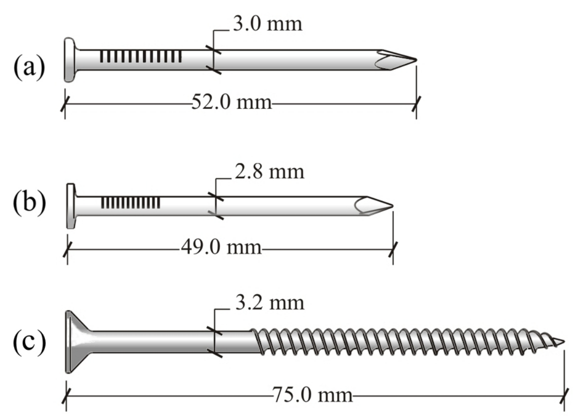

Figure 2: Fastener sizes and type. (a) and (b): bright wire nails, used for the OSB panel-to-frame fixing. (c): screws, used for the stud-to-beam connections.

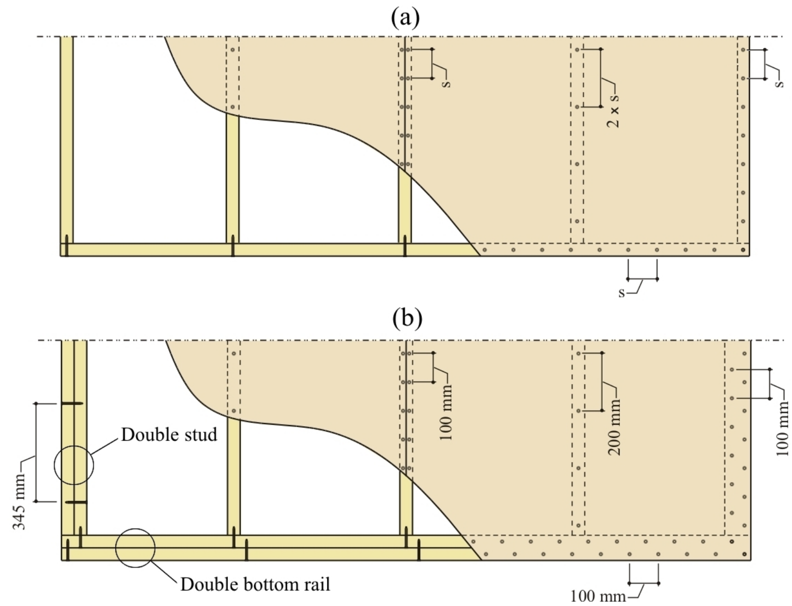

Figure 3: Wall specimen with: (a) standard frame, (b) frame with double end-studs and double bottom rail.

2.1. Wall specimens

All the wall specimens tested were assembled using C16 [19] white spruce timber with a cross-section of 44 mm × 95 mm, for the frame members, whilst 9 mm thick Oriented Strand Boards (OSB/3) [20] were used for sheathing. As reported in Table 2, two sizes of bright smooth wire nails were used for OSB panel-to-frame connections: 2.8 mm diameter × 49 mm long and 3.0 mm diameter × 52 mm long. Header beam and bottom rail were fixed to the studs by using 75 mm long screws with a smooth shank diameter of 3.2 mm (see Figure 2). For each specimen, the nail spacing of the sheathing panels along the intermediate studs was set at twice the perimeter nail spacing.

The effects of use of additional studs and bottom rails were examined by doubling studs at the leeward and windward sides of the wall specimens by screwing together two (44 mm wide × 95 mm deep) timber members at 345 mm centres. The panel-to-frame fixings along the double studs and double bottom rail were spaced at 100 mm on two staggered rows, effectively providing pairs of fasteners spaced at 100 mm (see Figure 3-b).

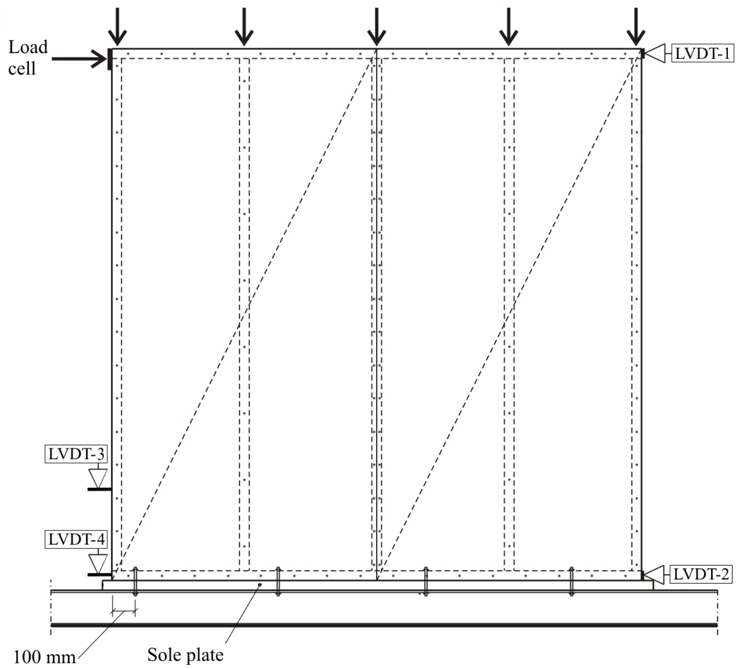

Figure 4: Raking test set up in accordance with BS EN 594:2011 [21].

2.2. Test set-up

The racking tests were carried out according to BS EN 594:2011 requirements [21]. With reference to Figure 4, a sole plate was positioned between the bottom rail of each wall specimen and the test rig base, and the bottom rail was fixed to the test bed by four 12 mm diameter bolts. The load was then applied by a load actuator at the top-left corner of the wall, whilst two linear transducers (LVDT-1 and LVDT-2) were used to take readings of the horizontal deformations.

The racking deformation of the wall (Δ_h) was calculated as the difference between the horizontal displacement of the header beam (LVDT-1) and the rigid body horizontal translation of the wall (LVDT-2). In order to avoid lateral movement of the wall specimens tested, a system of bracing and rollers was devised for the purpose.

2.2.1. Vertical load

The vertical load, where relevant, was applied by the use of a pressurised air-bag, sandwiched between two plywood panels, and located between the header beam of the wall specimen and the overlying loading rig cross-bar (see Figure 5).

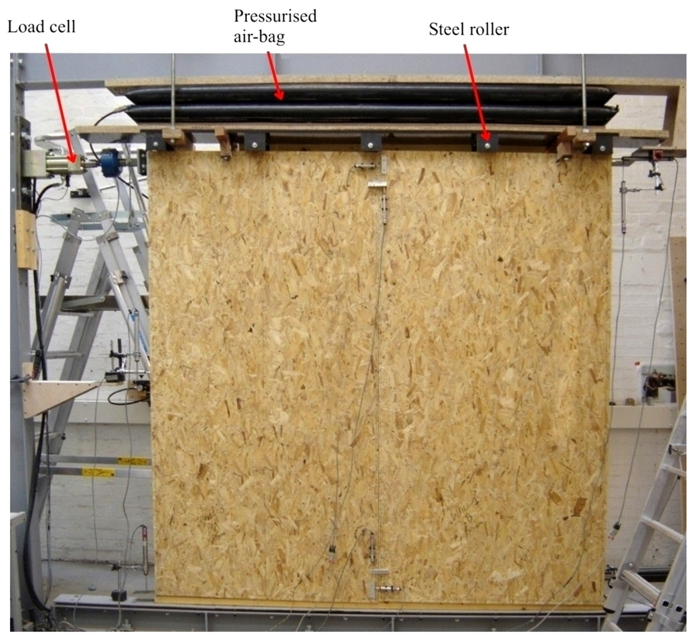

Figure 5: Application of vertical loading by air-bag device and steel rollers system.

Table 1: Moisture content and density values from tested walls.

Material	Average density [kg/m^3]	Average moisture content [%]
Timber – C16	375	13.0
OSB/3	591	5.5

To avoid frictional forces affecting the racking test results, the air-bag device was sat on steel rollers positioned close to top of each stud, hence simulating the path of vertical loading transferred to the wall from horizontal floor joists. The required air pressure was calibrated for different increments of total vertical loading.

2.3. Test series

Four series of tests, on wall specimens, all with constant height of 2.4 m, were carried out, totalling 17 wall specimen tests. A detailed description of each wall specimen, corresponding test result and test series, are given in Table 2.

Table 2: Wall specimens – summary of test series and test results.

Test ID	Wall length [mm]	Frame type	No. of studs	Vertical load [kN]	Nail size [mm]	Nail[a] spacings [mm]	Experimental results: strength [KN]	Experimental results: stiffness[b] [KN/mm]
I-1						50	23.13	1.647
I-2	2400	standard	5	0	2.8×49	100	19.79	0.708
I-3						150	13.10	0.408
II-1						50	40.72	1.774
II-2	2400	standard	5	25	3.0×52	100	30.18	1.483
II-3						150	21.46	1.430
III-1	300		2[c]				0.89	0.015
III-2	600		2				2.36	0.066
III-3	900	standard[d]	3[e]	0	2.8×49	100	3.06	0.162
III-4	1200		3				7.24	0.206
III-5	1800		4				9.08	0.358
IV-1	300		2				1.04	0.017
IV-2	600	double	2				3.53	0.059
IV-3	900	end studs &	3	0	2.8×49	100[f]	6.72	0.182
IV-4	1200	double	3				10.74	0.278
IV-5	1800	bottom rail	4				16.69	0.599
IV-6	2400		5				25.82	0.938

[a] Of the perimeter panel-to-frame connections.

[b] As from Eq. (10).

[c] Studs spacing = 300 mm.

[d] See Figure 6.

[e] Stud spacings = 600 + 300 mm.

[f] along two staggered rows, as shown in Figure 3-b.

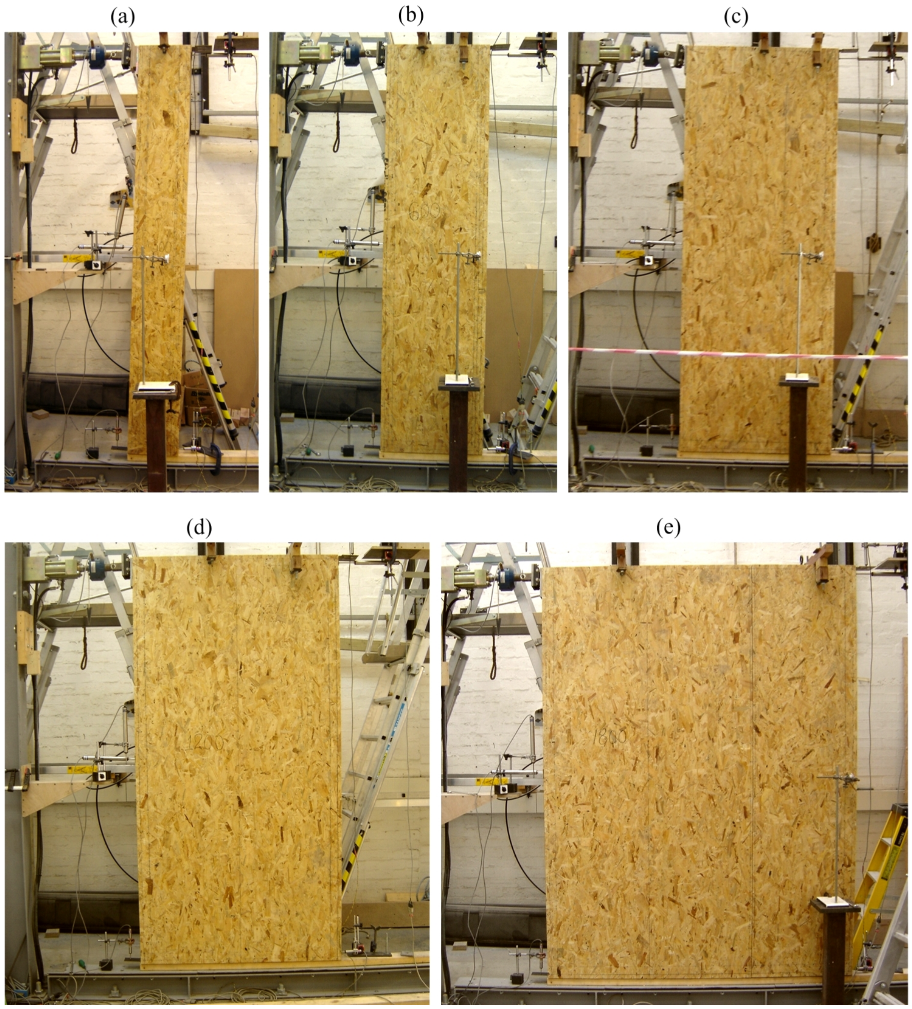

Figure 6: Racking test series III, as from Table 2, on timber unit walls with different length, L: (a) $L = 300$ mm, (b) $L = 600$ mm, (c) $L = 900$ mm, (d) $L = 1200$ mm, (e) $L = 1800$ mm.

2.4. Moisture content and density

Representative values of moisture content and density were determined from samples of the timber and OSB sheathing material used for the wall racking tests. The values are reported in Table 1.

2.5. PD 6693-1 method overview

The method described in PD 6693-1 is a semi-empirical approach mainly based on the development of a plastic theory model introduced by Källsner and Girhammar [22, 23] to predict the racking strength of partially anchored framed wall diaphragms. According to the PD method: when the panel-to-frame fasteners are fixed at uniform spacings, a lower bond value for the racking strength of the wall (indicated in the paper as $P_{h,max}$) can be determined by considering the panel-to-frame fastener strength per unit length, f_{pd}, cumulated along a certain length, ℓ_{eff}, and acting at the bottom of the wall:

$$P_{h,max} = f_{pd}\ell_{eff} \tag{1}$$

2.5.1. Fastener strength per unit length

The value of f_{pd} is derived by dividing the mean strength value of the panel-to-frame fasteners, $F_{v,mean}$, by the fastener spacing s:

$$f_{pd} = \frac{F_{v,mean}}{s} \tag{2}$$

As pointed out in [1], the reason for using a mean strength value in Eq. (2), instead of a characteristic 5-percentile value, is because when a significant number of fasteners are loaded in a line configuration (e.g. along the bottom of the wall) it is unlikely that all these fasteners will only achieve the minimum failure strength i.e. characteristic strength value. According to the PD 6693-1 method, the mean strength value for the panel-to-frame connections is derived from the characteristic (5-percentile) value, $F_{v,Rk}$, increased by a minimum of 20% (for $s = 50$ mm) up to a maximum of 30% (i.e. for $s = 150$ mm):

$$F_{v,mean} = (1.15 + s)F_{v,Rk} \tag{3}$$

In order for Eq. (3) to be valid, the value of s has to be expressed in m. For OSB panel-to-frame connections, the value of $F_{v,Rk}$ can be derived by following the EC5 procedure (based on the Johansen plastic model [24]) to determine the strength of laterally loaded connections formed using metal dowel fasteners. As all of the fasteners will be in single shear for all of the wall test configurations, the characteristic load-carrying capacity of the connection will be obtained from EC5 Eq. (8.6), and the critical mode of failure for both nail sizes and materials considered in this study, will be *failure mode (d)*:

$$F_{v,Rk} = 1.05\frac{f_{h,1,k}t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}t_1^2 d}}-\beta\right]+\frac{F_{ax,Rk}}{4} \tag{4}$$

in which:

- t_1 = thickness of the sheathing panel, in mm.
- d = nominal nail diameter, in mm.
- $f_{h,1,k}$ = characteristic embedment strength of the sheathing panel in N/mm^2, which for OSB panels is taken as equal to $65d^{-0.7}t_1^{0.1}$ (EC5 Eq. (8.22)).
- $\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$, with $f_{h,2,k}$ being the characteristic embedment strength, of the timber frame members, in N/mm^2, which is equal to $0.082\rho_k d^{-0.3}$ (EC5 Eq. (8.15)), with $\rho_k = 310$ kg/m^3 [25].
- $M_{y,Rk}$ = characteristic yield moment of the nail in Nmm, taken as equal to: $0.3 f_u d^{2.6}$, (EC5 Eq. (8.14)), and the wire tensile strength f_u, is taken to be 600 N/mm^2.
- $F_{ax,Rk}$ = withdrawal capacity of the nail, taken as the minimum value between that obtained from EC5 Eq. (8.24) and 60% of the first term in Eq. (4), i.e. in agreement with the requirement of EC5 clause 8.2.2.(2) for round nails.

The mean load carrying capacity, $F_{v,mean}$, for OSB panel-to-frame connections made with bright smooth wire nails, has been calculated from Eqs. (3) and (4). In addition, for the same type of connection, $F_{v,mean}$ has also been derived from experimental tests on OSB panel-to-frame connection samples. The test procedure used, together with the results, are briefly described in Appendix A and a summary of the $F_{v,mean}$ values is given in Table 3.

2.5.2. Effective anchoring length

Having derived the relevant values of f_{pd}, the remaining parameter to insert into Eq. (1) in order to obtain the theoretical racking strength of the wall, is the

Table 3: Load carrying capacity of the OSB panel-to-frame connection, $F_{v,mean}$.

Nail size [mm]	Nail spacing, s [mm]	$F_{v,meam}$ as from EC5[a] [N]	$F_{v,meam}$ as from tests[b] [N]
2.8 × 49	50	667	779
	100	694	
	150	722	
3.0 × 52	50	730	1256
	100	760	
	150	791	

[a]Eqs. (3) and (4) in this paper.

[b]See appendix.

effective anchoring length ℓ_{eff}, which is obtained from:

$$\ell_{eff} = -\frac{H}{\mu} + \left[\frac{H^2}{\mu^2} + L^2\left(1 + \frac{2M}{\mu f_{pd} L^2}\right)\right]^{0.5} \tag{5}$$

where H and L are the height and base length of the wall respectively; M is the stabilising moment at the leeward side of the wall, which, for the walls being tested, will equate to:

$$M = Q\frac{L}{2} \tag{6}$$

and Q is the total load in kN acting along the top of the wall:

The term μ in Eq. (5) is the ratio between the withdrawal capacity of the connections fixing the wall to the underlying structure per unit length (f_{ax}) and the panel-to-frame fastener strength per unit length (f_{pd}):

$$\mu = \frac{f_{ax}}{f_{pd}} \tag{7}$$

For values of strength ratio per unit length f_{ax}/f_{pd} greater than 1, μ must be set equal to unity. This is because when $f_{ax} > f_{pd}$, the failure condition will be dictated by the strength of the panel-to-frame connections. For all of the racking tests described in this paper, the base rail of the walls are anchored to the test rig basement by bolts (see section 2.2), and so $\mu = 1$.

Another validity requirement concerns the value of the effective anchoring length, which is subjected to the following inequality conditions:

$$\text{If } \ell_{eff} \text{ as from Eq. (5)} \quad \begin{cases} > L \Rightarrow \ell_{eff} = L \\ < 0 \Rightarrow \ell_{eff} = 0 \end{cases} \tag{8}$$

Finally, for walls formed using wood based panel material, in order to limit the racking deflection to an acceptable serviceability load condition, the empirical relationship given in clause 21.5.2.3 of the PD-6693-1 document must be met. The relationship has been rearranged to suit the format used in this paper, taking into account the type of walls being investigated, and is:

$$\frac{f_{d,pd}\ell_{1,eff}}{L} \leq 8\frac{L}{H} \tag{9}$$

where $f_{d,pd} = (k_{mod}f_{pd})/\gamma_M$. For the type of materials used in the wall and for the test programme undertaken under service class 1 conditions, the values for the modification factors are set according to the UK National Annex to EC5 [26] i.e. $k_{mod} = 1.0$ and $\gamma_M = 1.3$. The value for $\ell_{1,eff}$ is derived from Eq. (5) with f_{pd} being replaced by $f_{d,pd}$.

3. Results, Analysis and Discussion

The experimental load-displacement curves, obtained for the wall specimens tested, are shown in Figure 7. From these curves it has been possible to derive the variation of racking strength as a function of the nail spacings and wall length parameters (sections 3.1 and 3.2), enabling a comparison to be made between the experimental results and the values calculated by using the analytical procedure described in the PD 6693-1 method. The experimental load-displacement curves allowed also a quantitative investigation on how the variation of nail spacings and wall length affect the racking stiffness of the timber framed wall (section 3.3.1). The experimental values for the ultimate racking load and racking stiffness values are given in Table 2.

The analytical procedure described in section 2.5 has been used to compute the racking strength values of the tested unit framed walls as well as stiffness behaviour, and comparison with test results is provided in the following subsections.

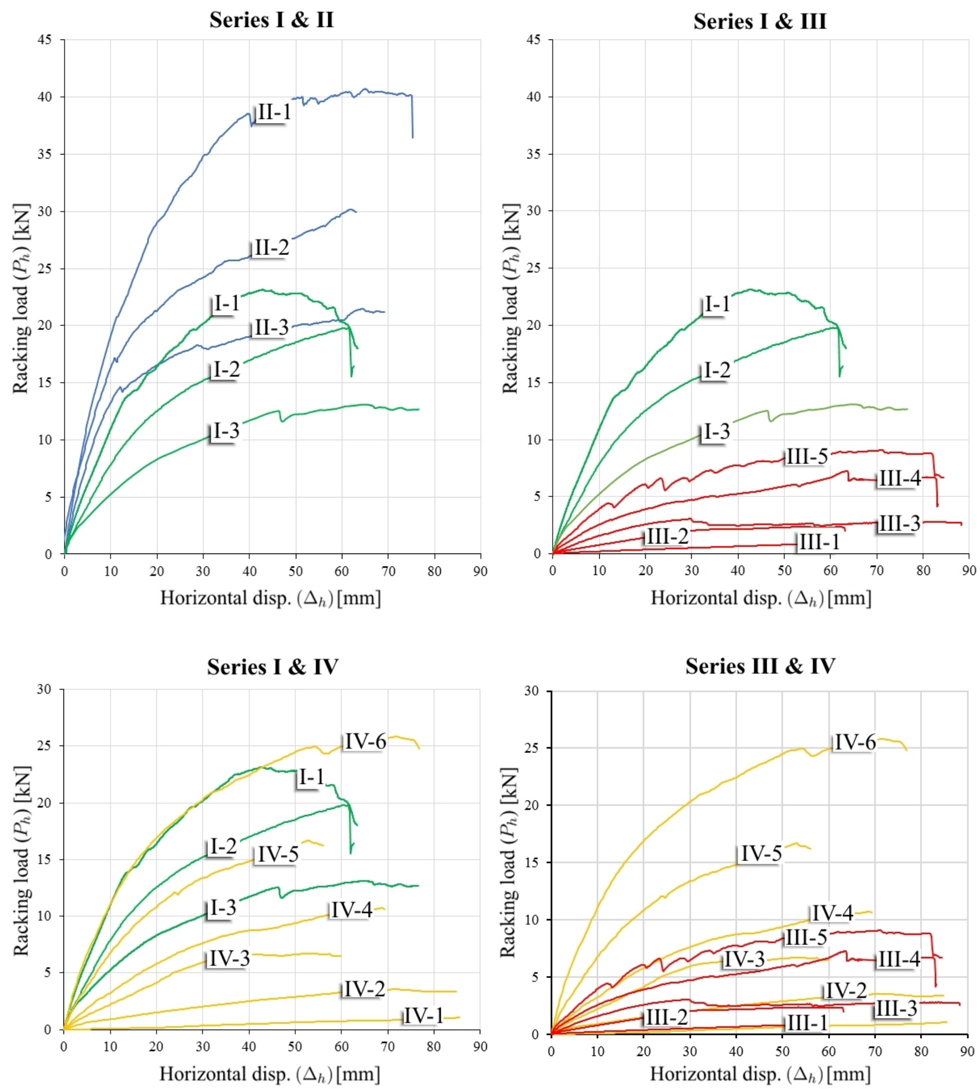

Figure 7: P_h-Δ_h curves and corresponding test ID, as given in Table 2.

3.1. Effect of nail spacings on the racking strength

Figures 8-a and 8-b show the variation of racking strength as a function of the panel-to-frame nail spacings, obtained respectively from tests on wall specimens without and with vertical loading, i.e. test series II and I (see Table 2). To allow comparison with the corresponding analytical functions (bold lines with circles for values based on $F_{v,meam}$ derived from test results; bold lines with diamonds for values based on $F_{v,meam}$ derived from EC5 design rules), the test values have been fitted with a linear function (dashed lines) such that $P_{h,max}(s) = \alpha s + \beta$. Values for the square of the multiple correlation coefficient, R^2, are given on Figure 8.

On this basis, it can be seen that, regardless of the nail spacing, the racking strength values predicted analytically (by Eq. (1)) follow a similar trend to those derived by tests, but are consistently lower. Also, the analytical values for function $P_{h,max}(F_{v,mean})$ with $F_{v,mean}$ derived from EC5 method (Eq. (4) in the paper), provide lower results than those obtained by using the value for $F_{v,mean}$ derived from tests. The difference between the two analytical curves is greater for racking strength results on walls formed using the larger diameter nails (3.0 mm × 52 mm) i.e. Figure 8-b, and this is very much influenced by the difference between the fastener strength values of the 2.8 mm and 3.0 mm diameter nails derived from the lateral strength tests (see third and fourth columns of Table 3). For connections made with 2.8 mm × 49 mm nails, the mean strength value ($F_{v,mean}$) obtained from tests is 8%-16% higher than $F_{v,mean}$ as obtained from EC5 calculations, and this difference rises to 58%-72% when looking at the mean strength of connections made with 3.0 mm × 52 mm nails.

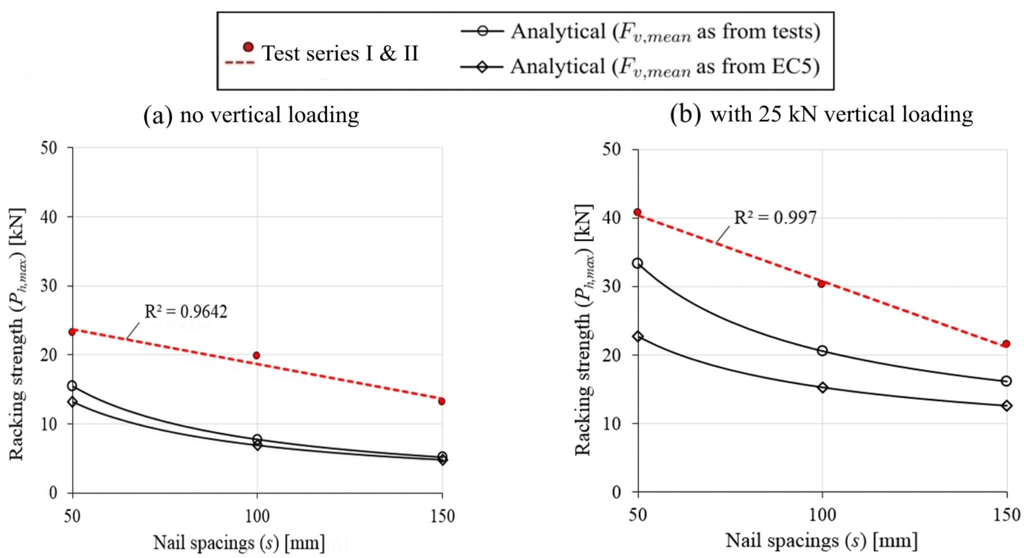

Figure 8: Wall racking strength as a function of the panel-to-frame fastener spacings (s). The experimental values are referred to: (a) test series I, i.e. walls assembled with 2.8 mm × 49 mm nails and without applied vertical load. (b) test series II, i.e. walls assembled with 3.0 mm × 52 mm nails and with 25 kN vertical load (see Table 2).

Since in the PD 6693-1 method the wall racking strength, $P_{h,max}$, is a function of the panel-to-frame fastener strength (see Eqs. (1-2)), it is not surprising that the analytical function $P_{h,max}(F_{v,mean})$, with $F_{v,mean}$ derived from EC5 calculations, provides lower values compared to the same function with $F_{v,mean}$ obtained from tests. This also explains the more pronounced difference between the two analytical racking curves when 3.0 mm $\times$ 52 mm nails are used to fix the panels to the frame (see Figure 8-b).

Making a comparison between the analytical results obtained using $F_{v,mean}$ from tests (round markers with continuous curve in Figure 8) and the experimental racking strength results (dashed curves), the following observations are made:

- With change in the nail spacing s, between 50 and 150 mm, the difference between the experimental and the analytical curves remains roughly constant at the 50 mm and 150 mm spacings. Although staggered downward, the analytical curves seem to effectively follow the variation of racking strength due to the different fastener spacings used. With reference to Figure 8-a, with s ranging from 50 mm to 150 mm, the experimental value of $P_{h,max}$ decreases from 23.13 kN to 13.10 kN (-10.03 kN) and the analytical value of $P_{h,max}$ decreases from 15.49 kN to 5.16 kN (-10.32 kN). Similarly, with reference to Figure 8-b, the experimental value of $P_{h,max}$ drops from 40.72 kN to 21.46 kN (-19.26 kN) and the analytical value of $P_{h,max}$ from 33.39 kN to 16.09 kN (-17.29 kN).

  In relative terms however, the analytical underestimation of racking strength increases with the increase of the nail spacing s. Referring to the test case with no applied vertical load (Figure 8-a): for s = 50 mm, the analytical function gives a racking strength that is -33% the corresponding experimental value, whilst for s = 150 this difference increases to -61%. A similar, but less pronounced difference, is found for the test case with 25 kN vertical load (Figure 8-b): at s = 50 mm the analytical raking strength is predicted to be -18% the corresponding experimental value, whilst for s = 150 the underestimation increases to -25%.

- The underestimation of the analytical function is much more pronounced, in both relative and absolute terms, for the test case without vertical applied load. For this case, $P_{h,max}$ is calculated on average to be -53% (9.3 kN) less than the test result (Figure 8-a). This compared to an average difference of –25% (–7.4 kN) for the test case subjected to 25 kN vertical load (see Figure 8-b).

A possible explanation to why the analytical function gives more accurate results when a vertical load Q is applied to the top of the wall, is provided as follows. In the analytical approach, in accordance with the requirements of Eqs. (6) and (5), the racking strength of the wall increases with the increase of the stabilising moment M it supports. This is a function of the wall head loading being supported, i.e. $M = QL/2$. Another contributor to the stabilising moment will be the resistance offered by the stud-to-beam rail connections at the windward end of the wall, which is ignored in the PD 6693-1 equations for a combination of practical and conservative reasons. However, in this analysis, whilst for $Q = 25$ kN, such a contribution only represents a small percentage of the stabilising moment, for the case where $Q = 0$ kN there will be a contribution to M entirely due to the withdrawal capacity of these connections, which is ignored in the analysis. This affects the results and will contribute to the reason why there is a different behaviour between loaded and unloaded test and analytical results.

As previously seen, the analytical racking strength function $P_{h,max}(F_{v,mean})$, computed with $F_{v,mean}$ obtained from EC5 method, provides lower results compared to the same function computed with $F_{v,mean}$ obtained from tests. With reference to Figure 8-a, with s ranging from 50 mm to 150 mm, the analytical value of $P_{h,max}$ (computed with $F_{v,mean}$ as from EC5 method) decreases from 13.25 kN to 4.78 kN (-8.47 kN). Similarly, with reference to Figure 8-b, the same analytical value drops from 22.68 kN to 12.57 kN (-10.11 kN).

3.2. Effect of wall length on the racking strength

Figure 9 shows the variation of racking strength as a function of the wall length, derived from tests on walls made with OSB sheathings fixed on a standard frame (test series III plus I-2) and OSB sheathings fixed on timber frames made with double end studs and double bottom rail (test series IV). The test values have been fitted with a power function such that $P_{h,max}(L) = \alpha s^{\beta}$ since a better fit of the experimental data is achieved, compared to a linear function.

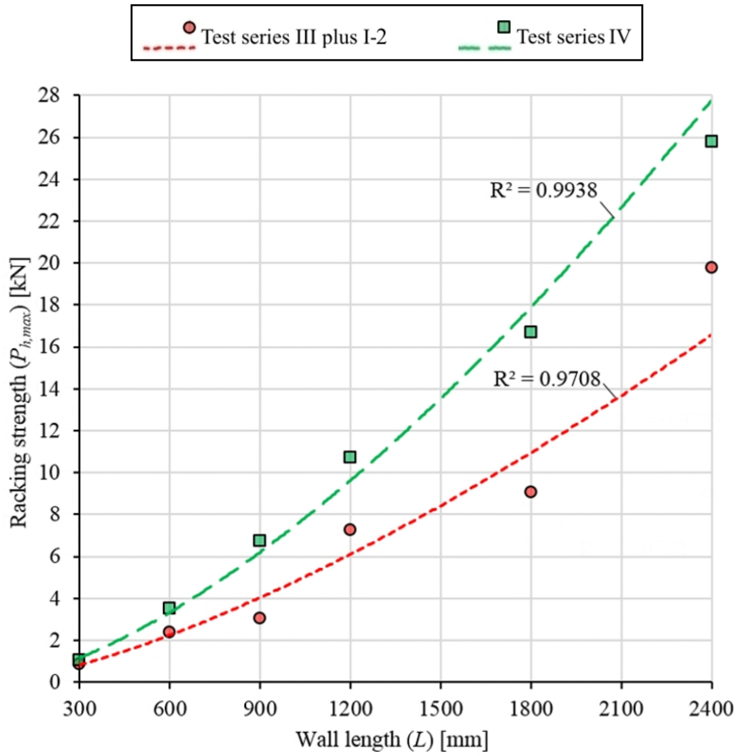

Figure 9: Experimental racking strength as a function of the wall length (L). The values are referring to walls made with a standard frame (test series III plus I-2) and walls made with frames assembled with double end studs and double bottom rail (test series IV), see Table 2.

The wall specimens made with a standard type frame have a racking strength of 0.89 kN for L = 300 mm up to 19.79 kN for L = 2400 mm. In comparison, the walls made with double studs and a double bottom rail are much stronger, with strength values ranging from 1.04 kN for L = 300 mm, up to 25.82 kN for L = 24000 mm (i.e. about 58% higher, on average). The reason for such a strength increase is primarily due to the use of a double row of fasteners along the perimeter of the wall (see Figure 3), rather than any strength contribution from the double end-studs and double bottom rail. Considering the cumulated lateral strength of two rows of fasteners at 100 mm spacings to be equivalent to two rows of fasteners at 100 mm spacing, a comparison of results can be made between wall test I-1 and IV-6: wall I-1 has a racking strength of 23.13 kN, which is only 10% lower than the racking strength of wall IV-6 (25.82 kN).

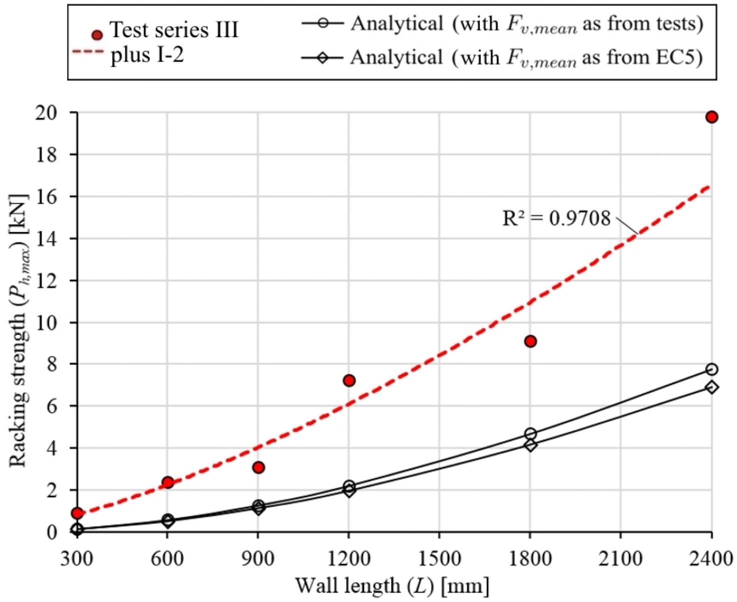

Figure 10: Experimental and analytical racking strength as a function of the wall length (L). The experimental values are referring to test series III plus I-2, i.e. walls made with sheathings fixed at 100 mm spacings on a standard frame. The fastener load carrying capacity, $F_{v,mean}$, required to compute $P_{h,max}$, has been derived both from tests (see appendix) and from EC5 procedure, i.e. Eqs. (3) and (4) in the paper.

In Figure 10 a comparison of racking strength results obtained from tests (test series III plus I-2), and strength values obtained analytically, based on tests and EC5 values, is shown. The experimental curve is derived from test results of walls assembled with 2.8 diameter $\times$ 49 mm long nails spaced at 100 mm, and with no vertical loading. As can be observed from the Figure, the analytical raking strength curves remain well below the experimental curve for the entire range (i.e. 300 mm $\leq L \leq$ 2400 mm). In particular, the relative underestimation increases as the wall length is reduced: for $L = 2400$ mm, the analytical racking strength is predicted between 6.90 (based solely on EC5) and 7.75 kN (based on EC5 using test values), i.e. about 65% and 61% less than the experimental value (19.79 kN). As the wall length reduces to 300 mm, the analytically predicted racking strength becomes about 80% lower than the corresponding experimental value of 0.89 kN.

3.3. Racking stiffness behaviour

For each tested wall specimen, the corresponding racking stiffness, R, has been evaluated in accordance with the requirement of BS EN 594:2011 [21] as follows:

$$R = \frac{0.4P_{h,max} - 0.2P_{h,max}}{\Delta_4 - \Delta_2} \tag{10}$$

in which Δ_4 and Δ_2 are the values of the wall deformation recorded respectively at 40% and 20% of the maximum racking load $P_{h,max}$.

The particular relationships investigated in regard to stiffness behaviour are covered in the following subsections.

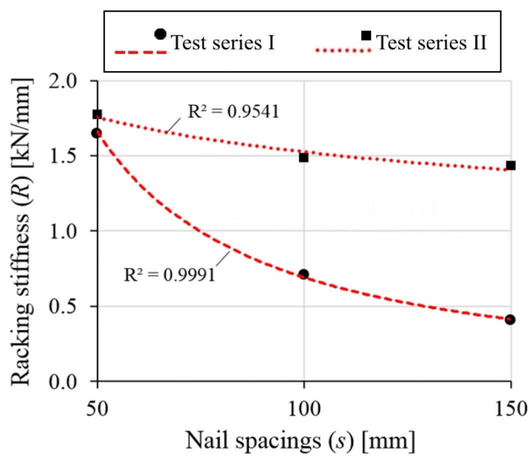

Figure 11: Racking stiffness as a function of the nail spacing (s). Values referring to test series I, i.e. walls without applied vertical load, and test series II, i.e. walls with 25 kN applied vertical load.

3.3.1. Effect of nail spacings on the racking stiffness

Figure 11 shows the variation of racking stiffness, R, as a function of the nail spacing s, obtained from tests on wall specimens without vertical load (test series I) and also with 25 kN vertical load (test series II), both walls being 2400 mm long. The racking stiffness, R, was derived from tests according to Eq. (10). As expected, the racking stiffness is enhanced as the nail spacing is reduced. For the case with 25 kN vertical load, R rises from 1430 N/mm (for s = 150 mm) to 1774 N/mm (for s = 50 mm) i.e. an increase of 23.8%. For the same wall without vertical loading there is a much steeper increase in racking stiffness, rising from

410 N/mm (for s = 150 mm) to 1647 N/mm (for s = 50 mm), corresponding to an increase of 300%. Also, at a nail spacing of 50 mm, the racking stiffness of the unloaded wall is approximately 93% of the loaded wall condition. From this it can be seen that the stiffness of unloaded walls is more greatly influenced by nail spacing than loaded walls of the same construction, and also that as the nail spacing reduces the stiffness is primarily influenced by the nail spacing rather than the vertical loading.

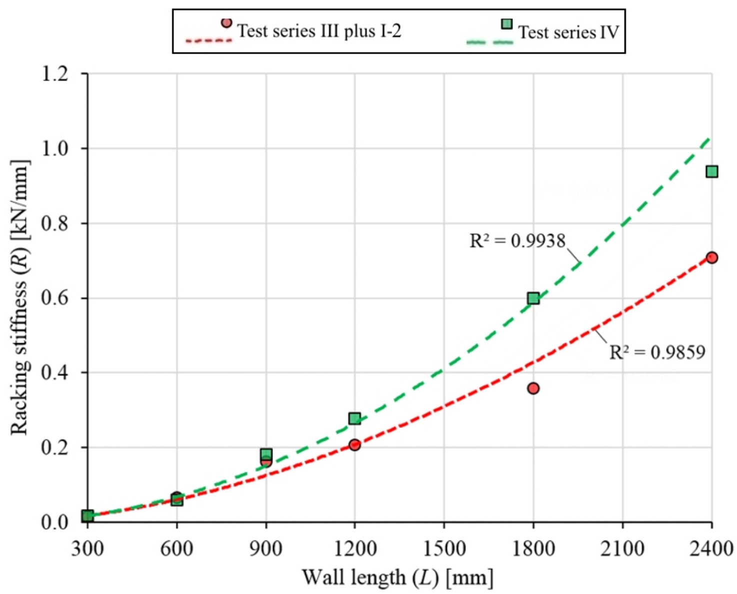

Figure 12: Racking stiffness as a function of the wall length (L). The values are referred to walls made with OSB panels fixed on a standard frame (test series III plus I-2) and OSB panels fixed on a timber frame made with double studs and bottom rail (test series IV). See Table 2.

3.3.2. Effect of wall length and frame construction on the racking stiffness

A plot of racking stiffness values, R, against the wall length, L, is shown in Figure 12. The Figure gives plots of wall specimens made with OSB sheathing panels fixed to a standard frames (test series III plus I-2), and wall specimens with sheathings fixed on frames made with double end studs and double bottom rails (test series IV). In line with the stiffness to nail spacing behaviour referred to in section 3.3.1, the racking stiffness, as well as the rate of increase in stiffness, increases with the length of the wall. For short walls (i.e. up to 900 mm) the increase in stiffness and rate of change of stiffness are approximately linear and despite the stiffer frame construction associated with the test series IV walls, the behaviour of both types of wall is similar. Above this wall length however, the stiffness values start to increase at a more rapid rate, and for the 2400 mm walls assembled with double studs and double bottom rails the racking stiffness is about 32% stiffer than the same length of wall constructed using the standard type of frame.

For the shorter walls the wall shear deformation per unit racking force will make a larger contribution than for longer walls as it is a function of the ratio of panel-height to panel-width. The factor will range from 8, for 300 mm long walls, to 1 for 2400 mm long walls. Therefore, for longer walls the lateral shear deformation of the wall panels becomes less significant and the major contribution to stiffness is the behaviour of the sheathing fasteners and the racking frame. The configuration of the fasteners is similar for both types of wall, however, from the test results, doubling up on the end studs and the bottom rails has made a significant contribution to stiffness behaviour.

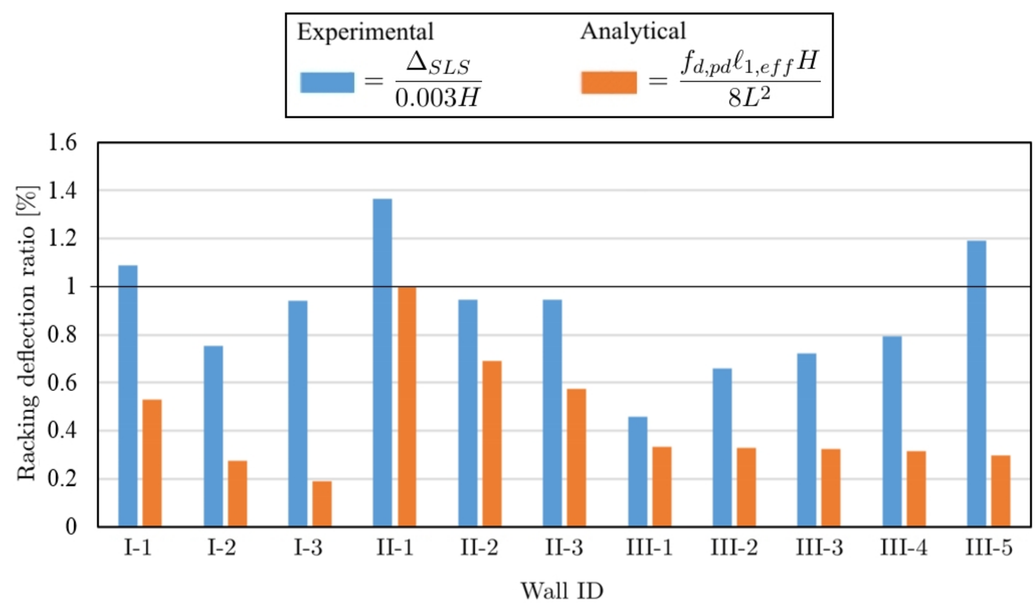

Figure 13: Comparison between the racking deflection limit ratios based on test results ($\Delta_{SLS}/0.003H \leq 1$) and PD 6693-1 rules.

3.3.3. Effect of PD 6693-1 rules on design strength and stiffness values

In the PD 6693-1 document, in order to limit the racking deflection of a wall, a stiffness criterion has been introduced and to suit the format used in this paper it has been re-arranged and is given in Eq. (9). In accordance with the functions used in PD 6693-1, this empirical relationship can be expressed in terms of the design racking load, P_{ULS}, of the wall at the Ultimate Limit State (ULS), where:

$$P_{ULS} = f_{d,pd}\ell_{1,eff} \tag{11}$$

enabling Eq. (9) to be rewritten as:

$$\frac{P_{ULS}H}{8L^2} \leq 1 \tag{12}$$

The value of the design racking load for each wall test has been calculated in accordance with the procedure defined in PD6693-1, with the $f_{d,pd}$ values derived using the values of the panel-to-frame fastener strength, $F_{v,mean}$ obtained by the application of the EC5 design procedure, given in Table 3. Inserting the relevant functions into Eq. (12) for walls I, II and III, a plot of the results is shown in Figure 13.

Since no limiting relationship for an acceptable value of racking stiffness is given in BS EN 594:2011, the deflection limit of 0.003 times the panel height, given in BS 5268-6.1:1996 [27], has been used as the limiting deformation that would be acceptable. It is also anticipated that this deflection limit will be incorporated into the next revision of the UK National Annex for BS EN 1995-1-1 as the maximum lateral deformation that will be permitted at the Serviceability Limit State (SLS), for such walls. Based on the test results, a plot of the ratio $\Delta_{SLS}/0.003H$ for walls I, II and III is given on Figure 13 to allow comparison with the empirical relationship for the limitation of displacement at the serviceability state given in PD6693-1, restructured as presented in Eq. (12). All walls tested were 2400 mm high, resulting in a deflection limit of $0.003H = 7.2$ mm. As the stiffness criteria relationship in equation Eq. (12) is based on characteristic design values, to obtain equivalent load values from the test curves, the test load results have been modified by a factor of 0.8, as given in Table 8 of BS 5268-6.1:1996. Also, to derive the deflection at the serviceability state, Δ_{SLS}, associated with the P_{ULS} design load, the value has been taken to be that obtained from the modified test results at a load of $P_{ULS}/1.5$.

From the Figure it can be seen that based on the above procedure, all walls will pass the stiffness criterion set by the PD6693-1. However, when comparing with the deflection limit criterion $\Delta_{SLS}/0.003H \leq 1$, walls I-1, II-1 and III-5 will fail. In all cases, the results from the PD6693-1 criterion indicate that the walls are generally well within the limiting value except for wall II-1, which is on the limit of acceptability. When analysed using the deflection limit approach, $\Delta_{SLS}/0.003H$, three walls fail (walls I-1, II-1 and III-5), and further three are close to the failure (I-3, II-2 and II-3) and in every instance this approach indicates there is a smaller margin against compliance than in the case where the PD6693-1 criterion is used. In practice, vertically loaded walls will be selected over unloaded walls to provide racking resistance to a structure and so the walls of particular interest in a stiffness comparison exercise are walls II-1, II-2 and II-3. For these three walls, the ratio of the experimental to analytical results is on average 1.45 and as the fastener spacing reduces the walls stiffness gets closer to the limiting stiffness condition, with wall II-1 exceeding the limit when based on the experimental approach.

4. Conclusions

The present work aimed to assess, by means of experimental tests, how the variation of some common parameters, such as fastener spacing and wall length, affect the racking behaviour of timber Platform framed walls, enabling evaluation of the accuracy of the formulae proposed in the design code to determine the racking strength and stiffness of the walls. In particular, the investigation has been focused on partially anchored racking walls, the most common method of construction adopted for timber framed walls in the UK. Consequently, the procedure described in the PD 6693-1 document, as recommended by the UK NA to EC5, has been adopted. From the analyses and test results described in section 3, the following conclusions are drawn:

- In general, the racking strength of the wall is more sensitive to variations in the fastener spacings when it is subjected to a vertical loading. Conversely, when the wall has no vertical loading, its racking stiffness becomes more sensitive to change in fastener spacings.
- The effect of panel-to-frame fastener spacing is more pronounced when the wall is subjected to an applied vertical loading. For example, the gain in

strength for walls without vertical loading, when the fastener spacing was reduced from $s = 150$ mm to 50 mm, was 76% compared to the increase of 89% for a similar wall under a vertical loading of $Q = 25$ kN.

- In the case of racking stiffness, for walls without vertical loading, the gain in stiffness was up to 300% when the fastener spacing was reduced from $s = 150$ to $s = 50$ mm. However, such gain in stiffness did not occur in similar walls when they were subjected to a vertical loading of $Q = 25$ kN, with stiffness increasing by only 24%.

- The comparison of the experimental results of the full-length (2400 mm) wall specimens, irrespective of their panel-to-frame fastener spacings (50 mm to 150 mm), with the results from the design code formulae, showed that on average the design code underestimated the racking strength by 25% for walls under vertical loading of $Q = 25$kN and by 54% for walls without vertical loading. Noting that the analytical model only provides a lower bound value for the racking strength of the wall, the most likely explanation why such an underestimation is greater for walls without applied vertical load, is due to the contribution to the stabilising moment, M in Eq. (6), due to the withdrawal capacity of the stud-to-beam connections.

- Compared to walls made with a standard type of frame, the use of double studs and double bottom rails provides (on average) an increase in racking strength and stiffness of about 64% and 37% respectively. Nonetheless, the enhanced racking capacity may be (solely) attributed to the use of increased number of panel-to-frame fasteners along the perimeter of the wall.

- Considering stiffness behaviour, all walls comply with the requirements of the empirical relationship given in clause 21.5.2.3 of the PD-6693-1 document. However, when deriving stiffness behaviour from the experimental results, i.e. using the $\Delta_{SLS}/0.003H$ approach, walls I-1, II-1 and III-5 fail. It is difficult to draw any general conclusions on the accuracy of the PD 6693-1 criterion, however, as the more important situation in practice will relate to the behaviour of walls that carry vertical loading, i.e. walls II-1, II-2 and II-3, the behaviour of these walls show that both approaches result in an increase in value as wall stiffness is increased and for the stiffest wall, II-1, the experimental result shows the wall will fail whilst the PD-6693-1 approach concludes it will pass. As acceptable stiffness behaviour has to be achieved in the design of racking walls, it is to be questioned that the empirical relationship given in equation PD6693-1 may require to be reviewed.

5. Acknowledges

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Appendix A. Panel-to-frame connection tests

In order to derive the relevant value of $F_{v,mean}$, a total of twelve panel-to-frame connection samples, each comprising four bright wire smooth nails, were tested.

Six samples were assembled using 2.8 mm diameter × 49 mm long nails, and a further six were assembled using 3.0 mm diameter × 52 mm long nails. As shown in Figure A.1, two different types of test set-up were considered. For each nail size, three connection samples were tested by loading the OSB panel towards its edge (to cover for possible edge splitting failure) and three more samples with the OSB panel loaded away from its edge. The strength value, $F_{v,max}$, obtained from each sample test divided by 4 (the No. of nails per sample) is reported in Table A.1, whilst the values of $F_{v,mean}$ reported in the fourth column of Table 3 were taken as the average of the $F_{v,max}$ values reported in Table A.1.

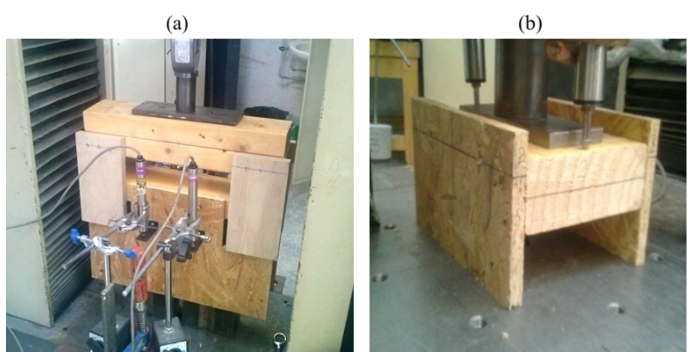

Figure A.1: Test set up to asses the strength of the panel-to-frame connections. (a): set up with the panel loaded towards its edge and (b): set up with the panel loaded away from its edge.

Table A.1: Summary of test results for the panel-to-frame connections.

Test No.	Nail size [mm]	$F_{v,max}$[a] [N]	Test set up[b]
1	2.8 × 49	732.0	(a)
2	2.8 × 49	789.7	(a)
3	2.8 × 49	827.2	(a)
4	2.8 × 49	720.7	(b)
5	2.8 × 49	835.5	(b)
6	2.8 × 49	770.2	(b)
Average =		779.2	
Standard deviation =		43.4	
Standard deviation / Average =		5.6 [%]	
7	3.0 × 52	1437.5	(a)
8	3.0 × 52	1287.7	(a)
9	3.0 × 52	1529.7	(a)
10	3.0 × 52	1090.2	(b)
11	3.0 × 52	1018.7	(b)
12	3.0 × 52	1174.2	(b)
Average =		1256.3	
Standard deviation =		182.3	
Standard deviation / Average =		14.5 [%]	

[a]Referring to the strength test result divided by the number of nails per sample (i.e. 4).

[b]As from Figure A.1.

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